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SEISMIC RESPONSE AND DESIGN OF RC STRUCTURES WITH PLAN-ECCENTRIC MASONRY INFILLS

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SUMMARY

The bidirectional response of a two-storey RC frame structure with two adjacent sides infilled is studied through shaking table tests and non-linear dynamic analyses. The pre-cracking stiffness of the infills is large enough to impose twisting of the infilled structure about the common corner of the two infilled sides, with predominant period close to that of translation of the symmetric bare structure in the two horizontal directions. Parametric analyses and test results show that the peak displacement components of the corner column of the two open sides are about the same as (or slightly less than) those of the bare structure under the same bidirectional excitation, but take place simultaneously. This simultaneity of peak local demands from the two components of the motion seems to be the only effect of plan-eccentric infilling that needs to be taken into account in the design of the RC structure. Despite their very high slenderness (height-to-thickness ratio of about 30), infill panels survive out-of-plane peak accelerations of 0·6*g* at the base of the structure or 1·3–1·75*g* at their centre. Copyright © 1999 John Wiley & Sons, Ltd.

KEY WORDS: infilled structures; plan-eccentric structures; RC structures; seismic design; shake table tests; torsional response

1. INTRODUCTION

Although there is still no consensus in the international community of structural earthquake engineering regarding the implications of masonry infills on the earthquake resistance and seismic safety of reinforced concrete buildings, there is wide agreement that negative effects are often associated with irregularities in the distribution of infills in plan and/or in elevation: plan-eccentric infills transform fairly symmetric and regular structures into torsionally unbalanced, while a reduction or full absence of the infills in a storey may cause concentration of inelastic deformation demands in its columns and development of a soft-storey mechanism. It seems though that our current understanding of the seismic response and performance of masonry-infilled structures is not enough for the development of detailed rules and procedures to

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account for the presence and spatial distribution of infills in the seismic assessment of existing structures or in the design of new ones.

Regarding the design of new structures for earthquake resistance, Part 1.3.2 (concrete) of Eurocode 8 (EC8) introduced design principles for masonry-infilled reinforced concrete buildings and detailed rules for the application of these principles.[1] Section 2.9.3.1 of Part 1-3 of EC8 refers to structures with strong irregularity of the infills in plan, mentioning specially the arrangement of the infills on two adjacent sides of the perimeter and requiring the analysis of a 3D model of such a structure, including a sensitivity study of the effect of the position and stiffness of the infills. No rules or guidance are given, though, regarding the models to be used for the infill panels in such an analysis. Rules of this type have been given recently in Annex 2 of the official instructions for the application of the Italian seismic code,[2] stating, among others, that infill panels can be modelled in an elastic analysis as a single diagonal strut with an equivalent axial stiffness, EA/L, equal to 10 per cent of the masonry elastic modulus, E_m, times the panel thickness, t.

A major problem with a design procedure in which infills are explicitly included in the model for the analysis, is that masonry infills, being traditionally non-engineered, have as-built properties which, at the design stage are almost impossible to estimate reliably and/or to specify, and at the construction stage are hard to control. Current standardization, national or international, of masonry units and mortar, is not enough for the characterization of the inelastic cyclic behaviour of masonry panels, with or without openings. Last but not least, during the service life of a structure the geometry of masonry infills may be subject to alterations beyond the control of the designer and of the building code. It seems, therefore, that before we see significant progress on the issues of knowledge and control of the behaviour and future state of as-built masonry infills, the only prudent approach for new structures is to design the structural system against conservative bounds for the seismic demands in the presence of infills, relying though as little as possible on quantitative information on the infill properties. This is more so for plan-eccentric infills, as the problem of the inelastic seismic response and design of torsionally unbalanced structures[3–11] is one of the most challenging and complex in structural earthquake engineering and the development of simple code-like, yet accurate-enough and generally applicable and widely accepted design rules for them is still pending. The situation is, of course, different regarding the seismic assessment and upgrading of existing buildings, not only because there the properties and layout of as-built infills is given, but also because the engineer is more interested in identifying and utilizing the beneficial effects of infills on the global seismic performance and safety of such buildings, than in protecting the structural system from their potentially adverse effects.

The present paper refers to reinforced concrete (RC) structures which are symmetric and torsionally balanced in both horizontal directions, but may develop lateral-torsional coupling due to a strongly eccentric layout of infills in plan. It focuses on a special case often quoted,[1,12–14] explicitly or implicitly, as the most extreme and potentially adverse plan-eccentric infilling: that of infills arranged only on two adjacent sides of the plan. As it is often encountered at the corners of building blocks, such a layout is more common than that of infills on solely one side. The main question addressed herein for this layout of the structure and its infills is whether and to what extent the seismic demands on RC elements are larger than in the bare structure under the same (bi-directional) ground motion. Another issue considered on the side is that of the coupled in-plane and out-of-plane response and safety of infill panels under bi-directional excitation. As the out-of-plane response of infill panels is induced by the mass and inertia of the panels themselves under transverse excitation, this issue can be studied experimentally only through bi-directional shaking table testing. To investigate these questions, the present authors have

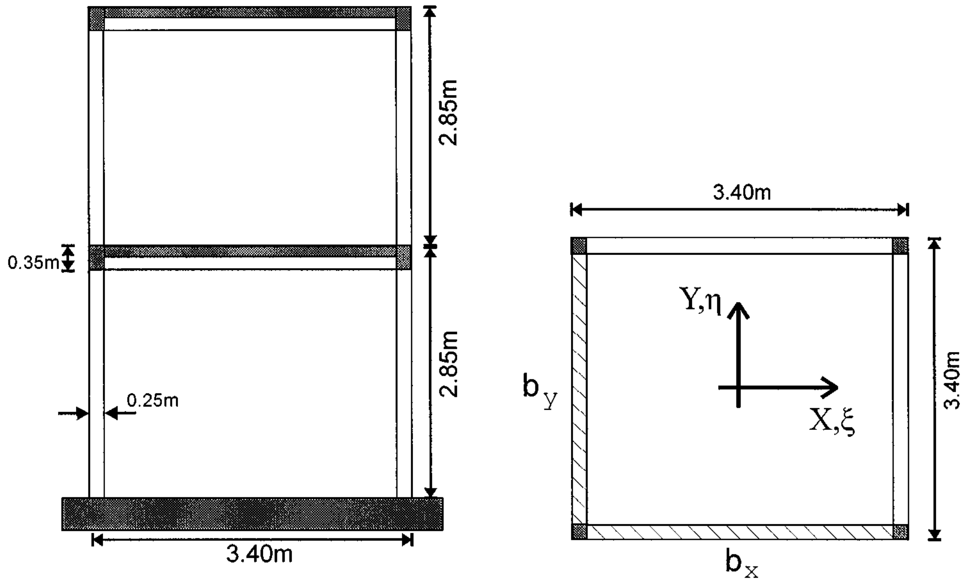

Figure 1. Geometry of two-storey structure

conceived, designed and performed a bidirectional shaking table test on a two-storey RC frame with one bay in each direction, having the two adjacent sides infilled in both storeys and the other two open. This paper explains the rationale behind the design of the test and the specimen, presents the most important experimental and analytical findings and draws conclusions regarding the seismic response and design of irregularly infilled RC structures on the basis of the results of the tests and of the accompanying parametric non-linear dynamic analyses.

2. TEST STRUCTURE AND SETUP

The two-storey test specimen, shown in Figure 1, has a storey height of 2·85 m, one 3·4 m wide bay in each direction, 0·25 m-square columns, beams 0·25 m wide by 0·35 m deep and 0·14 m thick slabs. It was designed with a scale factor of 1·0, according to Eurocode 8 Part 1–3·2, as a Ductility Class M (Medium) bare structure (implying a behaviour or force-reduction factor q or R of 3·75), for a design peak ground acceleration of 0·2 g and a design spectrum which is flat up to a period of 0·6 s. The fundamental period, based on uncracked gross sections, is 0·3 s, i.e. in the constant spectral acceleration range, giving a base shear coefficient of 0·135.

The design gave everywhere the minimum reinforcement: 8 bars in each column and 3 bars at the top and bottom of each beam, all of 12 mm diameter and grade S500 steel. With this minimum reinforcement the test structure possesses significant overstrength against the design ground motion. Normally for bare structures designed according to Eurocodes 2 and 8 there is an overstrength factor ranging between 1·5 and 2·0, due to the difference between the design values of material strengths and the actual strengths, and another due to the reduction in stiffness and force

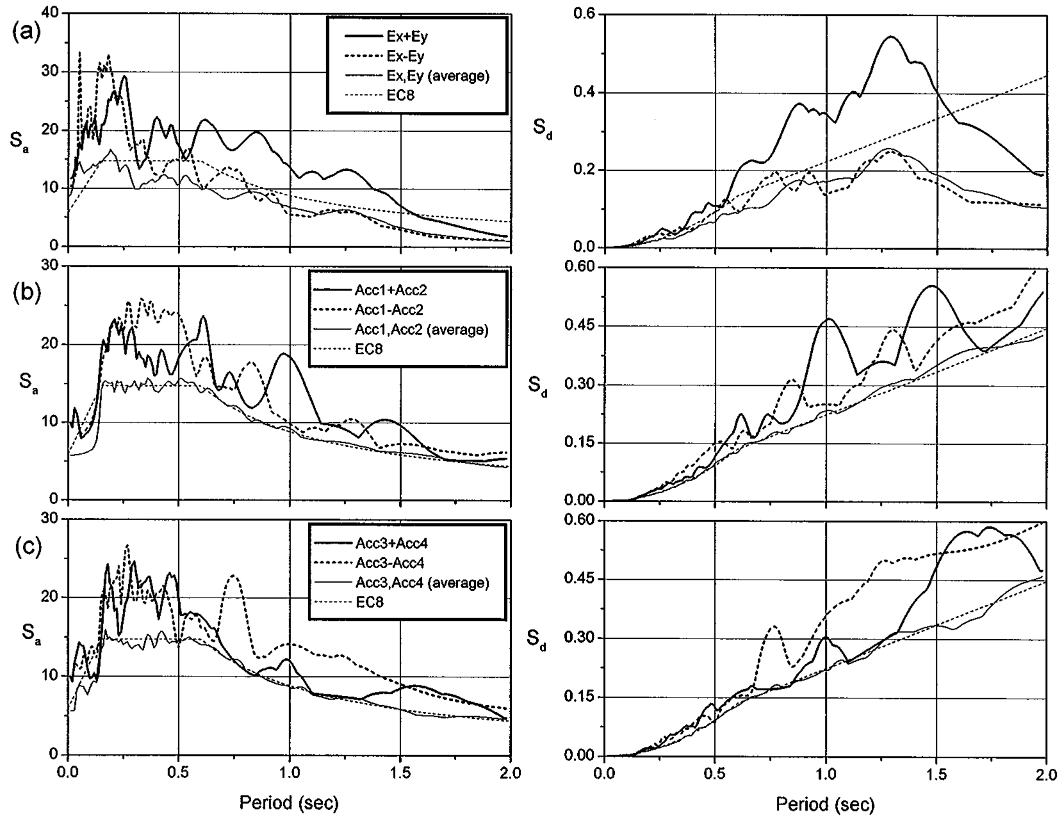

Figure 2. 5 per cent damped acceleration and displacement spectra for components of bidirectional motion and for their sum and difference; motions used in: (a) test; (b) and (c) in parametric analyses

demands with concrete cracking.[15] In the present case, this second overstrength factor is estimated to be around 1·6, giving an aggregate overstrength factor of around 3·0. To remove the effect of this overstrength, the test structure was subjected to simultaneous horizontal ground motion components with an effective peak acceleration three times that of the design motion (0·6 g instead of 0·2 g). Two 10 s-long acceleration time histories were synthesized for use in the test, approximately fitting the 5 per cent damped elastic spectrum for which the structure was designed, up to a period of 1·3 s. Longer period components were filtered out, to comply with the $\pm 0{\cdot}1$ m and $\pm 0{\cdot}5$ m/s displacement and velocity limits of the shaking table. The average acceleration and displacement spectra of the two motions, denoted here by E_x and E_y, are shown in Figure 2(a). Pre-test non-linear dynamic analyses of the response to such motions has shown that, due to the fact that reinforcement is everywhere controlled by the minimum requirements and not by strength demands under the design seismic action, inelastic demands in the test would had been limited. For this reason, it was decided to test the structure with a live load of 5 kN/m^2, instead of the 2 kN/m^2 for which it was designed, which corresponds to an increase in the total mass of 41 per cent.

Infilling of two adjacent sides was specified, with a thickness of 115 mm at the bottom storey or of 80 mm at the second storey. Due to their high slenderness ratio ($2{\cdot}5/0{\cdot}08 = 31$), infills in the top storey are vulnerable to damage and out-of-plane expulsion under simultaneous in-plane and out-of-plane response.

A companion bare specimen was tested to the same bidirectional base excitation. Both specimens were tested on the MASTER shaking table of the ISMES establishment in Bergamo (I).[16]

3. THEORETICAL BACKGROUND AND RATIONALE OF THE TEST

3.1. Fundamental considerations

Conception of the test was guided by the following considerations: The stiffness of uncracked infill panels is so large, that in the elastic range of the response of the infilled system the centers of stiffness and of rotation of the infilled structure coincide with the common corner of the two adjacent infilled sides and the first two modes of vibration of the two-storey irregularly infilled structure are rotational about that corner.

Let's consider a bare multistorey structure, rectangular in plan with dimensions b_x and b_y in directions X and Y and with distribution of mass and elastic stiffness which is at all levels symmetric in plan with respect to both axes X and Y. For such a distribution of properties in 3D, the main features of the elastic translational response in directions X and Y can be captured by idealizing the bare structure as an equivalent elastic SDOF system in X and Y. The equivalent mass, m_e, lateral stiffness, k_{xe}, k_{ye}, etc. properties of this system can be established assuming a height-wise variation of lateral drifts, e.g. linear. If the origin of the global co-ordinate system OXY is at the centre of the plan, dimensionless coordinates, $\xi = 2x/b_x$ and $\eta = 2y/\mathrm{b}_y$, are defined with values ± 1 at the corners. Then let us assume that the lateral stiffness of the elastic bare structure in directions X and Y and the mass, being functions of ξ and η, are distributed in plan in the following way:

$$k_x(x, y) = k_{x\mathrm{e}} f_{kx}(\xi, \eta) \tag{1}$$

$$k_y(x, y) = k_{y\mathrm{e}} f_{ky}(\xi, \eta) \tag{2}$$

$$m(x, y) = m_{\mathrm{e}} f_m(\xi, \eta) \tag{3}$$

Assuming further that:

$$f_m(\xi, \eta) = f_m(\eta, \xi) \tag{4}$$

and taking into account that $\int_A \xi f_m(\xi, \eta)\,\mathrm{d}A = \int_A \eta f_m(\xi, \eta)\,\mathrm{d}A = 0$ and $\int_A f_m(\xi, \eta)\,\mathrm{d}A = 1$ (similarly for $f_{kx}(\xi, \eta)$ and $f_{ky}(\xi, \eta)$), in the structure with the two adjacent sides infilled the effective rotational inertia with respect to the corner of the infilled sides, let us say at $\xi = -1$, $\eta = -1$, is

$$J_{\theta\mathrm{e}} = \int_A m(x, y)\left[\left(x + \frac{b_x}{2}\right)^2 + \left(y + \frac{b_y}{2}\right)^2\right]\mathrm{d}A = \frac{m_e(b_x^2 + b_y^2)}{4}\left(1 + \int_A \xi^2 f_m(\xi, \eta)\,\mathrm{d}A\right) \tag{5}$$

Similarly the effective rotational stiffness $k_{\theta e}$ of the elastic bare structure with the two sides at $\xi = -1$, and $\eta = -1$ infilled, with respect to the corner $\xi = -1$, $\eta = -1$, is

$$k_{\theta e} = \int_A \left[k_x(x,y)\left(y + \frac{b_y}{2}\right)^2 + k_y(x,y)\left(x + \frac{b_x}{2}\right)^2 \right] \mathrm{d}A$$
$$= \frac{k_{xe} b_y^2}{4}\left(1 + \int_A \eta^2 f_{kx}(\xi,\eta)\,\mathrm{d}A\right) + \frac{k_{ye} b_x^2}{4}\left(1 + \int_A \xi^2 f_{ky}(\xi,\eta)\,\mathrm{d}A\right) \tag{6}$$

Under the assumption that the plan-wise distribution of lateral stiffness of the elastic bare structure in X and Y satisfies the condition

$$f_{kx}(\xi,\eta) = f_{ky}(\eta,\xi) \equiv f_k(\xi,\eta) \tag{7}$$

and denoting $f_{kx}(\xi,\eta)$ simply by $f_k(\xi,\eta)$, equation (6) yields

$$k_{\theta e} = \frac{k_{xe} b_y^2 + k_{ye} b_x^2}{4}\left(1 + \int_A \xi^2 f_k(\xi,\eta)\,\mathrm{d}A\right) \tag{8}$$

Considering then that the first mode of the elastic structure with the two sides infilled consists of twisting about their common corner, the circular frequency of this motion is

$$\omega_\theta = \sqrt{\frac{k_{\theta e}}{J_{\theta e}}} = \sqrt{\frac{\omega_x^2 b_y^2 + \omega_y^2 b_x^2}{b_x^2 + b_y^2}} \sqrt{\frac{1 + \int_A \xi^2 f_k(\xi,\eta)\,\mathrm{d}A}{1 + \int_A \xi^2 f_m(\xi,\eta)\,\mathrm{d}A}} \tag{9}$$

in which $\omega_x = \sqrt{k_{xe}/m_e}$ and $\omega_y = \sqrt{k_{ye}/m_e}$ are the circular frequencies of the elastic bare structure in its fundamental translational modes in directions X and Y.

If the plan-wise distribution of the mass or the elastic stiffness is uniform, the corresponding value of $\int \xi^2 f(\xi,\eta)\,\mathrm{dA}$ is 2/3; if the mass or the stiffness is concentrated at, and evenly distributed among the four sides, the value of the integral is $\frac{5}{6}$, while if it is lumped at the corners the integral equals 1. Usually both stiffness and mass have similar distributions in plan (e.g. uniform) and the second square root in equation (9) is equal to 1·0. In the extreme cases in which the mass is uniformly distributed in plan and the stiffness is concentrated along the perimeter or lumped at the corners, the second square root in equation (9) equals 1·05 or 1·095, respectively. It follows then that the predominant period T_θ of twisting about the corner of the two infilled sides is intermediate between those of translation of the bare structure in the two orthogonal directions, T_x and T_y, or sometimes slightly shorter. For equal lateral stiffness in the two horizontal directions, $k_{xe} = k_{ye}$, T_θ is equal to $T_x = T_y$ if the stiffness is uniformly distributed in plan, or to 95 per cent and 91 per cent of $T_x = T_y$, if it is concentrated at the perimeter or at the corners, respectively.

The equation of motion of the equivalent elastic SDOF system representing the infilled structure, for the angle of rotation u_θ about the common corner of the two infilled sides due to a unidirectional acceleration $a_{gx}(t)$ in direction X is

$$\ddot{u}_\theta + 2\zeta\omega_\theta\dot{u}_\theta + \omega^2 u_\theta = -\frac{a_{gx}}{J_{\theta e}}\int_A m(x,y)\left(y + \frac{b_y}{2}\right)\mathrm{d}A = -\frac{0{\cdot}5 m_e b_y a_{gx}}{J_{\theta e}}\int_A f_m(\xi,\eta)\,\mathrm{d}A = -\frac{0{\cdot}5 m_e b_y a_{gx}}{J_{\theta e}} \tag{10}$$

Then the maximum value of u_θ, $u_{\theta,\max,x}$ is equal to the spectral displacement S_{dx} at T_θ, times $0{\cdot}5 b_y m_e / J_{\theta e}$

$$u_{\theta,\max,x} = \frac{2S_{dx}(T_\theta)}{b_y(1 + (b_x/b_y)^2)(1 + \int_A \xi^2 f_m(\xi,\eta)\,dA)} \tag{11}$$

i.e. for uniformly distributed mass, to $1{\cdot}2 S_d b_y/(b_x^2 + b_y^2)$. The peak displacement components in directions X and Y at the corner of the two open sides are equal to $b_y u_{\theta,\max,x}$ and $b_x u_{\theta,\max,x}$, and for nearly square plan ($b_x \approx b_y$) they are less than the X-displacement of the bare structure, $S_d(T_x)$. The resultant displacement will be maximum at the corner of the two open sides, in a direction normal to the diagonal and equal to

$$\delta_{\max,x} = u_{\theta,\max,x}\sqrt{b_x^2 + b_y^2} = \frac{2S_{dx}(T_\theta)\cos\alpha_x}{1 + \displaystyle\int_A \xi^2 f_m(\xi,\eta)\,dA} \tag{12}$$

with $\cos\alpha_x$ denoting the direction cosine of the diagonal with respect to the X-axis.

If a unidirectional acceleration time history $a_{gy}(t)$ is applied in direction Y, term $-b_y a_{gx}$ in the right-hand side of equation (10) is replaced with $b_x a_{gy}$. The maximum resultant displacement $\delta_{\max,y}$ will be at the same point and direction but will be given by an equation similar to equation (12), with y instead of x. If the motion is bidirectional with two independent components $a_{gx}(t)$ and $a_{gy}(t)$, the term $-b_y a_{gx}$ in the right-hand side of equation (10) is replaced with $b_x a_{gy} - b_y a_{gx}$. If the two motions conform to the same spectrum, the maximum resultant displacement can be estimated by combining those due to the two components according to the SRSS rule as $(\delta_{\max,x}^2 + \delta_{\max,y}^2)^{1/2}$, yielding, with the spectral displacements S_{dx} and S_{dy} being at the same period T_θ and hence equal

$$\delta_{\max,x+y} = \frac{2S_d(T_\theta)}{1 + \displaystyle\int_A \xi^2 f_m(\xi,\eta)\,dA} \tag{13}$$

i.e., for planwise uniformly distributed mass: $\delta_{\max,x+y} = 1{\cdot}2 S_d(T_\theta)$. Its components in directions X and Y are equal to $\delta_{\max,x+y}\cos\alpha_y$, $\delta_{\max,x+y}\cos\alpha_x$, respectively, and are generally less than those of the bare structure under the same bidirectional motion, which are equal to $S_d(T_x)$ and $S_d(T_y)$, respectively.

As mentioned in the Introduction, arrangement of the infills only on two adjacent sides in plan is often considered,[1,12–14] as the most adverse plan-eccentric infilling. To compare with this case, we also consider here the same elastic structure with only one side infilled, namely the side corresponding to $y = -b_y/2$ i.e. to $\eta = -1$. If the infill is considered as practically rigid, the centre of rotation about which twisting will take place is at the middle of the infilled side, i.e. at $\xi = 0$, $\eta = -1$. For excitation along the X-axis, equation (10) still applies, but $J_{\theta e}$ and $k_{\theta e}$ are given by

$$J_{\theta e} = \int_A m(x,y)\left[x^2 + \left(y + \frac{b_x}{2}\right)^2\right]dA = \frac{m_e}{4}\left[(b_x^2 + b_y^2)\int_A \xi^2 f_m(\xi,\eta)\,dA + b_y^2\right] \tag{14}$$

$$k_{\theta e} = \int_A \left[k_x(x,y)\left(y + \frac{b_y}{2}\right)^2 + k_y(x,y)x^2\right]dA = \frac{k_{xe}b_y^2 + k_{ye}b_x^2}{4}\int_A \xi^2 f_k(\xi,\eta)\,dA + \frac{k_{xe}b_y^2}{4} \tag{15}$$

In the most usual case in which both the mass and the elastic stiffness are uniformly distributed in plan, the circular frequency of the twisting motion is

$$\omega_\theta = \omega_x \sqrt{\frac{4 + (\omega_y/\omega_x)^2 (b_x/b_y)^2}{4 + (b_x/b_y)^2}} \tag{16}$$

giving again $\omega_\theta = \omega_x = \omega_y$ if the lateral stiffness is the same in X and Y. If the planwise distribution of mass is uniform but the lateral stiffness is either lumped at the corners or concentrated along the sides, the period of the twisting motion, T_θ, of a square structure with the same lateral stiffness in the two directions ($b_x = b_y$, $k_{xe} = k_{ye}$) is equal to 88 or 94 per cent respectively of $T_x = T_y$, i.e. it is slightly shorter than when two adjacent sides are infilled instead of one (corresponding values: 91 or 95 per cent of $T_x = T_y$).

For $J_{\theta e}$ given by equation (14), the maximum value of u_θ due to unidirectional excitation in direction X is

$$u_{\theta,\max,x} = \frac{2S_{dx}(T_\theta)}{b_y\left(1 + \left(1 + \left(\frac{b_x}{b_y}\right)^2\right)\int_A \xi^2 f_m(\xi,\eta)\,dA\right)} \tag{17}$$

which is always greater than that of equation (11) for the structure with the two sides infilled. The plan-wise maximum resultant displacement is

$$0{\cdot}5 u_{\theta,\max,x} b_y \sqrt{4 + \left(\frac{b_x}{b_y}\right)^2}$$

i.e., for the usual case of planwise uniformly distributed mass

$$\delta_{\max,x} = \frac{3S_{dx}(T_\theta)}{\sqrt{4 + (b_x/b_y)^2}} \tag{18}$$

which is significantly larger than that of equation (12) for two sides infilled and uniformly distributed mass ($\delta_{\max,x} = 1{\cdot}2S_{dx}(T_\theta)\cos\alpha_x$).

The conclusion is that, for elastic response to unidirectional excitation, the translational in-plane displacements of the two open sides of a nearly square doubly symmetric structure with two adjacent sides infilled will have about the same predominant period and a peak value about 60 per cent of that of the bare structure under the same excitation. The corner column which is free on all four sides will experience about 60 per cent of the peak displacement of the columns of the bare structure, but simultaneously in X and Y. For simultaneous application along X and Y of two components conforming to the same smooth response spectrum, the response of the elastic infilled structure will still consist primarily of a twisting motion about the common corner of the two infilled sides, and the free column will experience simultaneous peak displacements in X and Y, about equal to $0{\cdot}6\sqrt{2} = 0{\cdot}85$ of those of the bare structure under the individual components of the bidirectional motion.

The considerations above refer to elastic RC elements and to nearly rigid infills. The in-plane stiffness of uncracked infills is so much higher than that of the RC members, that, as long as they do not crack, the infills can indeed be considered as practically rigid. As far as the RC members are concerned, it has been demonstrated through several hundreds of non-linear dynamic analyses of the response of bare RC frame structures to excitations at and above the design ground motion, that peak displacements and deformations are, on the average, approximately

equal to those predicted from a 5 per cent-damped elastic spectrum, provided that all RC members are considered with their secant stiffness at yielding at both ends in antisymmetric bending: i.e., with $EI = (M_y/\theta_y)L/6$,[17,18] in which M_y/θ_y is the average ratio of yield moment to the corresponding chord rotation at yield at the two ends for both directions of bending, and L the member length. A key factor in such a generalization of the equal displacement rule to multistorey frames is that, when fully cracked and at incipient yielding, such systems have predominant period in the falling (velocity-controlled) branch of the acceleration spectrum. On the basis of these findings, the reasoning and the simple quantitative results above for an equivalent SDOF system, considered either bare or fully infilled on two adjacent sides, can be extended to bare or infilled RC frames responding well into the inelastic range of their RC members.

3.2. Application to the test structure

In the test structure, with the infill properties inferred from tests on wallettes[15] (namely: cracking stress τ_{cr} equal to 0·4 and 0·43 MPa in the 0·115 and 0·08 m thick walls of the 1st and 2nd storey; shear modulus G equal to 2·45 and 1·56 GPa, respectively; ultimate shear strength equal to $1{\cdot}3\tau_{cr}$), the centre of stiffness of the elastic structure (computed considering the secant stiffness of RC members at yielding of both ends and the stiffness GA/H of uncracked infills) is at a distance of 0·01 and 0·02 m from the axis of the 1st and 2nd storey corner column, and the centre of resistance (computed on the basis of the ultimate moments at column top and bottom and of the ultimate strength of infills) at distances thereof of 0·69 and 0·73 m, respectively. For elastic response of the infills, therefore, the centre of rotation is indeed at the corner of the two infilled sides.

Moreover, eigenvalue analyses, with the properties of the RC members and infills considered in the above calculation of the centre of stiffness and with masses lumped at the corners of the slabs, yield translational periods of the bare structure $T_x = T_y$ equal to 1·01 s and fundamental period of twisting of the infilled structure T_θ equal to 1·0 s, i.e. in accordance with the result of equation (9) for the distribution of stiffness and mass considered in the analysis.

For a square structure ($b_x = b_y = b$) with the same lateral stiffness in both directions, application of a bidirectional base excitation, $a_{gx}(t)$ and $a_{gy}(t)$, to the infilled version is mathematically equivalent to application of a unidirectional excitation equal to the algebraic sum or difference of the two unidirectional components, depending on the orientation of the excitations relative to the two infilled sides. The reason is that the right-hand side of equation (10) for bidirectional excitation is $0{\cdot}5(b_x a_{gy} - b_y a_{gx})m_e/J_{\theta e}$ and for $b_x = b_y = b$ it becomes $0{\cdot}5(b_{gy} a_{gx})m_e/J_{\theta e}$. For the two excitation signals E_x and E_y generated specially for the shaking table test, elastic spectral values due to the sum of the two components are (beyond a period of 0·6 s) almost double and those due to the difference are (beyond a period of 0·3 s) about equal to those of the individual components (Figure 2(a)). For the two pairs of synthetic accelerograms used in the parametric analyses of Section 5, denoted by Acc.1 & Acc. 2 and Acc.3 & Acc. 4, all of which conform with the Eurocode 8 elastic spectrum for all periods, spectral values for the algebraic sum or difference of the two components are on the average about 25 per cent higher than those of the individual components, regardless of the sign (Figure 2(b) and (c)). At the translational or twisting predominant period of the test structure with RC members considered with their secant stiffness at yielding of both ends in antisymmetric bending and with uncracked infills ($T_x = T_y \approx T_\theta \approx 1{\cdot}0$ s), spectral values for the sum or the difference of the motion components E_x and E_y for the test are 1·9 or 0·8 times the

average spectral value of the individual components at $T = 1{\cdot}0$ s; for the two pairs of components used in the parametric analyses, the sum of Acc.1 & Acc.2, or Acc.3 & Acc.4 produces at $T = 1{\cdot}0$ s spectral values, respectively equal to 2·05 or 1·4 times the average of those of the individual components, whereas the corresponding values for the difference of Acc.1 & Acc.2, or Acc.3 & Acc.4 are 1·1 & 1·65 (despite the large dispersion of the six values at $T = 1{\cdot}0$ s, their average is close to $\sqrt{2}$, as the SRSS rule would give). As orientation of the two components relative to the infilled sides in the test and in the parametric analyses is such that it is their sum rather than the difference that produces the twisting about the corner, it is the values of 1·9 for the test and 2·05 or 1·4 for Acc.1 & Acc.2 or Acc.3 & Acc.4 respectively, which are of interest.

As the planwise distribution of masses in the test or in the analysis is such that the integral in the denominator of equation (11) is equal to 0·642 or to 1·0, respectively, giving a corresponding value of $u_{\theta,\max,x+y}$ equal to $0{\cdot}61 S_{\mathrm{d},x+y}/b$ or $0{\cdot}5 S_{\mathrm{d},x+y}/b$, in the test or in its numerical simulation peak displacement components at the free corner are estimated as $1{\cdot}9 \times 0{\cdot}61 = 1{\cdot}15$, or $1{\cdot}9 \times 0{\cdot}5 = 0{\cdot}95$, respectively, times the average of the peak translational displacements of the bare structure under the two unidirectional excitations. In the non-linear parametric analyses of the response to the two pairs of unidirectional motions, peak displacement components at the free corner column are estimated as $2{\cdot}05 \times 0{\cdot}5 = 1{\cdot}02$ for Acc.1 & Acc.2, or as $1{\cdot}4 \times 0{\cdot}5 = 0{\cdot}7$ for Acc.3 & Acc.4, times the corresponding average unidirectional values of the bare frame.

Inelasticity of the RC members is not expected to affect significantly the conclusions above. The non-linearity of the response of the infills is expected to reduce the peak displacements of the corner column, due to: (a) shifting of the instantaneous centre of rotation outside the plan of the structure, because of post-cracking softening of the infills; (b) channeling of some of the input vibration energy into hysteretic energy dissipation in the infills or even into the out-of-plane vibration of the infills themselves. The features of the displacement response regarding predominant frequency and direction should not be affected, though, by the non-linear behaviour of the infills.

4. NON-LINEAR DYNAMIC RESPONSE ANALYSES AND SHAKING TABLE TEST RESULTS

The non-linear dynamic response of the two-storey test structure to the imposed bidirectional input motion is numerically simulated, within the framework of pre-test calculations and of post-test model validation studies.

A member-type, one-component, lumped-inelasticity model is used in the non-linear response analyses. The model employs a bilinear (end)moment–chord rotation relationship in monotonic loading and modified-Takeda type of hysteresis rules. It has been demonstrated,[19] through comparisons with test results on RC members subjected to various simultaneous load histories at their two ends, that this simple and numerically stable member model performs overall much better than much more sophisticated alternatives, e.g. with distributed inelasticity along the member and with fiber discretisation at the section level. For columns the model is applied independently in the two directions of bending, without coupling between them and neglecting the effect of the variation of axial force on flexural strength and deformation. A key feature of the model is that the elastic stiffness EI is established as $(M_y/\theta_y)L/6$, from the end moment and the chord rotation, M_y and θ_y, at yielding in antisymmetric bending (as average of the four values of EI at the two ends for negative or positive bending), accounting for full participation of the slab

reinforcement, up to a distance equal to one-quarter of the beam span L or the mid-distance of beams, to the tension flange of the beams. (This effective slab width is normally considered to correspond to the expected transmission of tensile forces in the slab reinforcement to the beam-column joint, through bar anchorage within the transverse beam and torsion in the beam itself). At the structure level, P–Δ effects are included and the damping due to sources other than post-yield hysteresis is considered to be of the Rayleigh type, with values of parameters such that a damping ratio of 5 per cent is obtained at the first natural period of the elastic bare structure and at half this value. Details and validation of the modelling through comparisons with quasistatic or pseudodynamic test results on full-scale structures or subassemblies, as well as with predictions of very detailed fiber models, are given elsewhere.[15,20]

The in-plane mechanical behaviour of infill panels is modelled through a pair of diagonal compression-only struts, the force–deformation law of which in monotonic loading reproduces the elastic stiffness GA_w/H, the cracking strength, $A_w\tau_{cr}$, and the ultimate strength, $1{\cdot}3\ A_w\tau_{cr}$, of the infill panel (A_w and H denote the horizontal cross-sectional area of the panel and its clear height, while G_w and τ_{cr} are its shear modulus and strength as measured on square wallette specimens in diagonal compression) and is characterized by a secant stiffness to ultimate strength equal to the elastic stiffness of the equivalent strut according to Reference 21. The shape of the hysteresis rules reproduces well the experimental behaviour in cycling loading and the values of the associated parameters are chosen so that an equivalent viscous damping ratio around 20 per cent is obtained in the first full post-cracking cycle, or about 5 per cent in subsequent cycles.

Infill panels subjected to out-of-plane inertia forces due to the transverse component of the seismic excitation, are modeled as equivalent elastic SDOF systems, with stiffness and ultimate strength computed through a two-way membrane action model.[22] This out-of-plane ultimate strength, upon attainment of which a panel is considered to collapse by out-of-plane expulsion, is taken to decrease linearly with the maximum-ever in-plane shear distortion of the panel. Details about the infill in-plane and out-of-plane models, their calibration and the estimation of their parameters from simple tests on wallettes, are given elsewhere.[23,24]

Pre-test simulations of the response of a companion bare frame give satisfactory agreement with measured floor accelerations and with floor displacements estimated thereof by double integration, as far as the waveform and the frequency content is concerned. However, they overestimate measured peak accelerations by about 60 per cent, while they predict peak displacements estimated from the measured accelerations within ± 20 per cent. Fourier analyses of the predicted and the measured accelerations or displacements agree in that the predominant period of the non-linear response is at about 1·1 s in both horizontal directions, i.e. 9 per cent longer than the calculated value 1·01 s for the elastic bare structure but much longer than the period of 0·3 s computed for the uncracked bare frame and used in its design. The bare frame suffered significant spalling of the concrete cover at the corners of the four column bases, as well as through-depth cracking and concrete spalling in most first floor beams at the column face. This damage is in fair agreement with damage index values from the analysis around 75 per cent for the column bases and about 20 per cent for the ground storey beams, with lower predicted values elsewhere.

The infilled test structure was subjected twice to the same bidirectional input motions: the two consecutive tests, termed Test 1 and 2, do not differ in frequency content of the results, but Test 2 produced larger displacements (difference in peak values of 15 per cent), as the result of the cracking and the damage inflicted by Test 1. It is the test on the virgin specimen, Test 1, which is considered herein as the basis for comparisons.

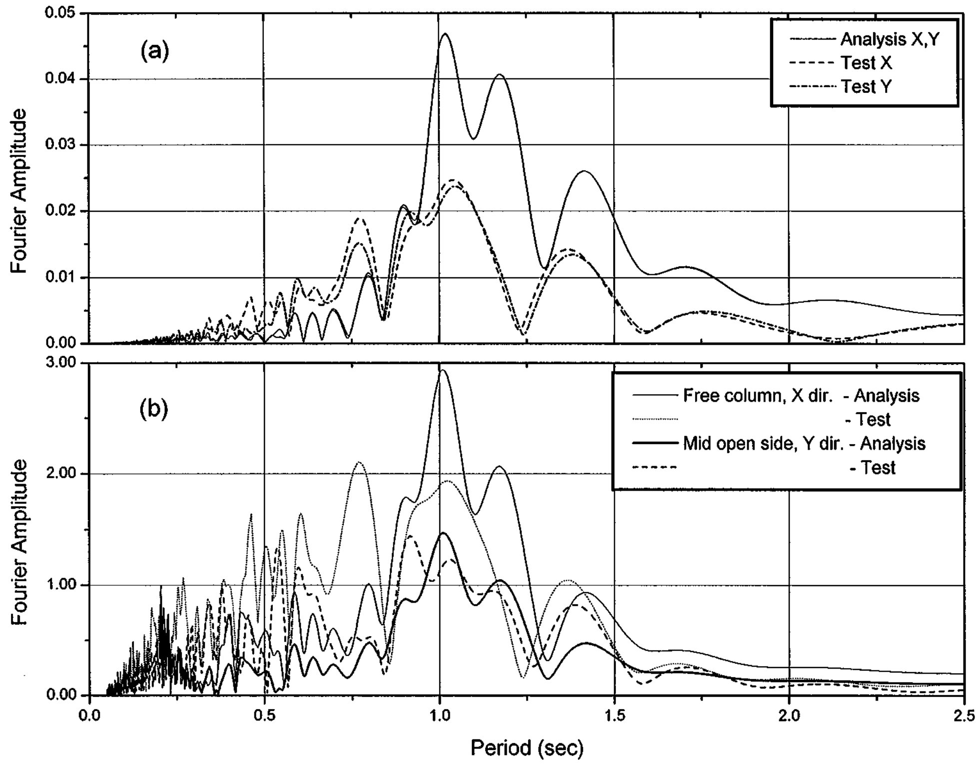

Figure 3. Fourier spectra of measured and calculated: (a) accelerations; (b) displacements; at top of infilled structure

Of special interest among the test results of the infilled structure are the out-of-plane accelerations at the centre of each one of the four infill panels. Computed and especially measured out-of-plane accelerations time histories exhibit frequencies of 10–25 Hz, which characterize the panel out-of-plane vibration. The predicted peak acceleration values at the centre of each panel are 3·4*g* at the top floor and 1·2*g* at the ground floor (corresponding average values over the panel of 2 and 0·7*g*), while the peak values measured there are 1·3 and 1·75*g*, respectively (corresponding average values over the panel are estimated equal to 0·75 and 1·0*g*). The measured peak values at the centre are below the response accelerations of about 4*g* at the top floor or 23*g* at the ground floor required for out-of-plane expulsion according to the model in Reference 22.

Fourier analysis results for the computed and the measured displacement time-histories in directions *X* and *Y* at the centre of the top floor are compared in Figure 3(a). Figure 3(b) compares the Fourier analysis results of computed and measured top accelerations in direction *X* at the free column and in direction *Y* at the middle of the open side. The predominant period of the computed response of the infilled structure is 1·0 s, i.e. as computed and measured in the bare frame, while that of the measured is about 1·05 s. This confirms the validity of equation (9).

Table I lists the peak positive or negative values of accelerations at the six points where accelerations were measured on the infilled test structure, along with the corresponding peak calculated values. The overall agreement between analysis and test is considered good, not only in the frequency content (Figure 3) but in the peak values as well.

Table I. Measured and computed peak accelerations (in g's)

Location and direction	Analysis		Test	
	Positive	Negative max	Positive	Negative
1st storey corner, X	8·17	− 8·47	5·70	− 9·49
1st storey corner, X	9·53	− 8·99	9·99	− 9·23
1st storey corner, Y	6·10	− 7·41	5·36	− 7·80
2nd storey corner, X	9·39	− 10·54	9·21	− 11·40
2nd storey corner, X	12·29	− 12·81	14·34	− 13·10
2nd storey corner, Y	9·35	− 8·68	8·90	− 8·06
Average	9·14	− 11·24	8·91	9·85

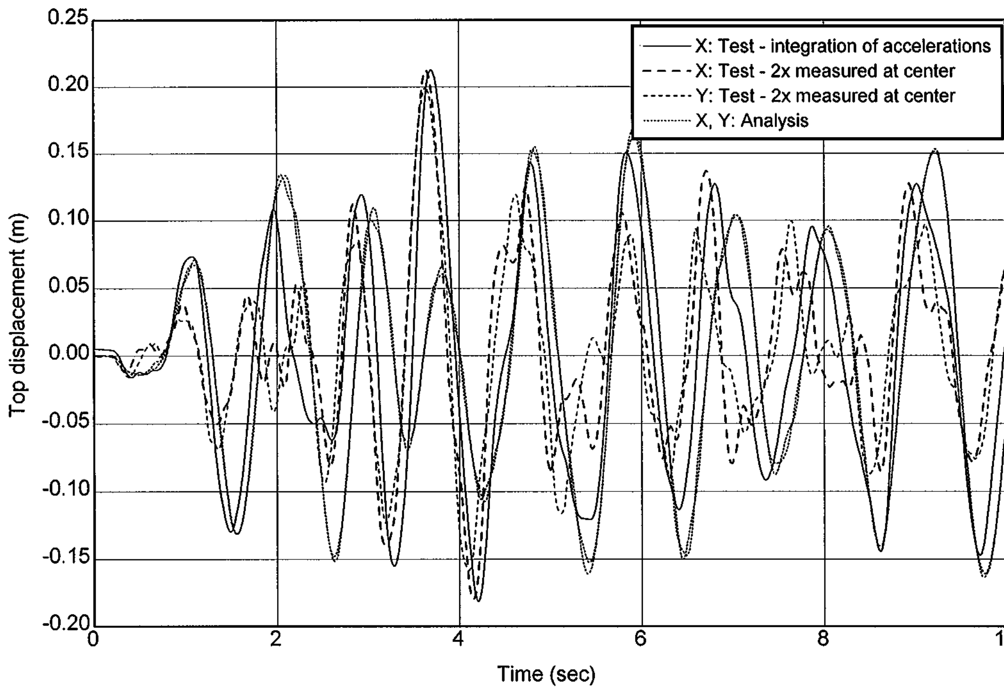

Figure 4. Free column top displacements of infilled structure: analysis vs. estimated test results

Figure 4 shows the top displacement response of the free column in the infilled test structure under bidirectional excitation, as computed and as inferred from the tests. Experimental displacements in direction X (solid line) were obtained by double integration of X-accelerations measured at the corner, while experimental displacements in both X and Y were also estimated by doubling the displacements measured at the centre of the top floor. The equality of computed displacements in X and Y, imposed by the twisting about the common column of the infilled sides, is evident. Indeed the instantaneous centre of rotation, computed from the peaks of the calculated

displacement response, is in both storeys very close to the corner column of the two infilled sides. There is discrepancy, however, between the X and Y displacements measured at the centre, as well as between the X-displacement at the free column inferred from that measured at the centre and the X-displacement estimated from the accelerations measured at the corner. Considering these discrepancies between measured values, the overall comparison of computed and measured displacements at the corner is considered satisfactory.

Measured peak displacements at the free corner of the infilled structures do not exceed much the corresponding peak values in the bare frame: in the infilled structure peak test values estimated from the X-accelerations or from the X and Y displacements at the centre are 21 or 20 mm in X or Y, while in the bare structure peak displacements estimated from measured accelerations are 20 or 17 mm in X or Y. Computed peak displacements are 16 mm in X and Y for the infilled structure and 16 or 19 mm in X or Y for the bare.

Regarding damage, the infilled structure at the base of the free corner column and the ground storey beams framing into this column suffered less damage than any of the columns and beams of the bare specimen. This is consistent with the predictions of the non-linear analyses, according to which the damage index at the base of the corner column is about 65 per cent (v. 75 per cent in the bare structure) and in the ground storey beams of the open sides around 12 per cent (v. 20 per cent in the bare structure).

The conclusion that peak seismic deformation demands and damage in the infilled structure do not exceed those in the bare frame subjected to the same bidirectional excitation, is confirmed by the measurements of rotations between the face of the foundation beam and sections of the columns 61 mm above. These rotations, compared in Figure 5(a) and (b), include the effect of bar pull-out from the anchorage in the foundation, and are derived from measured displacements with peak values of over 6 mm. If normalized to the gage length of 61 mm, these displacements yield tensile strains in the reinforcement which are possibly in excess of its elongation capacity. However, with a yield penetration length within the anchorage zone estimated around 140 mm, peak strains in the reinforcement are only about 3 per cent, which is consistent with the lack of bar buckling under the several load cycles imposed by the test. Of special interest are the mean axial elongations measured in the tests between the base and the section 61 mm above and are shown in Figure 5(c). They demonstrate a large axial elongation accompanying development of rotation peaks in either direction, as well as a ratcheting extension leading to a permanent elongation of the column. This form of axial-lateral coupling is typical of cyclic inelastic flexure under an average axial compression which produces in the columns a low axial load ratio,[3,25] as is presently the case. Interestingly column axial elongations in the infilled structure, be it peak values or residual ones, are lower than in its bare companion, especially in the columns of the two infilled sides. This disputes the claim made often that axial deformations and forces in columns of infilled frames are larger than those of bare frames, because the infill panel presumably transforms the frame into a composite shear wall with the columns acting as tension and compression chords.

5. SENSITIVITY STUDIES OF THE NON-LINEAR DYNAMIC RESPONSE

Numerical simulations were performed, of the response of the two-storey test structure to the test bidirectional input motions E_x and E_y and to two other pairs of horizontal ground motion components, Acc.1 & Acc.2 or Acc.3 & Acc.4, this time conforming to the smooth elastic response

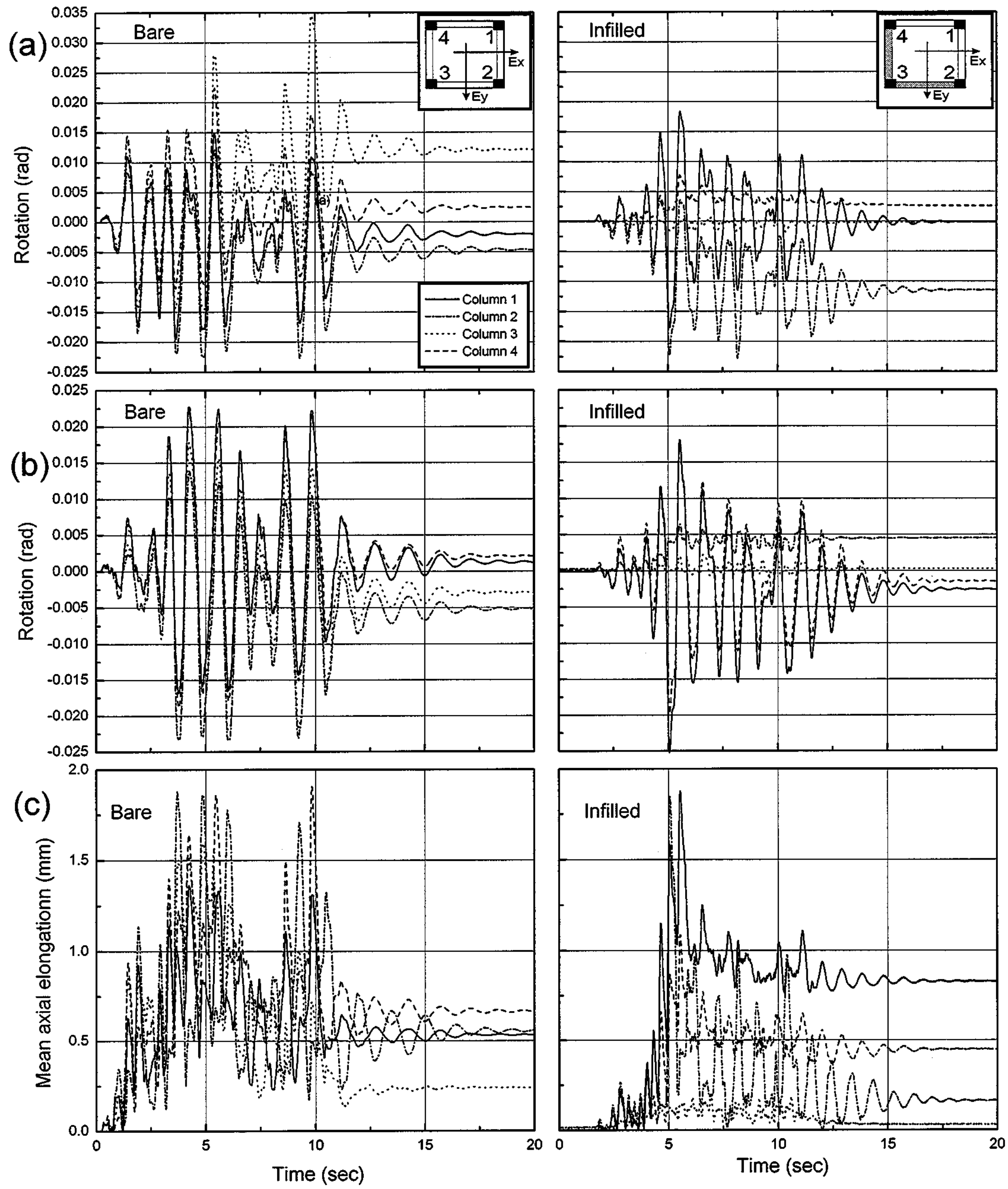

Figure 5. Rotations and mean axial elongations measured between base and column section 61 mm above: (a) rotations in plane XZ; (b) rotations in plane YZ; (c) axial elongations

spectrum over the full range of periods (see Figure 2(b) for the average spectra for Acc.1 and Acc.2 and Figure 2(c) for that of Acc.3 and Acc.4). The infilled structure considered in these parametric analyses differs from the test structure only in that the infills of the second storey are taken the same as in the first.

In a first series of parametric analyses the ordinates of the infill force–deformation law (and hence both the strength and the stiffness of the infills) are increased or decreased by a factor of 2·5, 5 or 10, relative to the base case of the strength and stiffness measured on wallettes for the purpose of the tests. The increase in stiffness and strength by a factor of 2·5, 5 or 10, does not change the centre of stiffness, but shifts the coordinates of the centre of resistance by 0·31, 0·47 or 0·57 m towards the corner, with respect to its location in the base case at 0·81 m from the centre in each horizontal direction. The reduction in stiffness and strength by a factor of 2·5, 5 or 10 shifts the co-ordinates of the centre of stiffness by only 0·01, 0·03 and 0·08 m from the centre in each horizontal direction, but shifts those of the centre of resistance towards the centre by as much as 0·35, 0·57 or 0·71 m, respectively. In other words, as long as the infills remain uncracked, the present variation of their properties does not affect materially the eccentricity, whereas if they crack and approach ultimate strength, the eccentricity and the associated torsional response are greatly reduced with the reduction of their strength and stiffness.

These parametric analyses, performed under three pairs of bidirectional motions, all scaled to a PGA of 0·6g, show little sensitivity of the response to this wide variation of infill properties. The infills crack only in the base case and in the cases with reduced strength and stiffness, but develop limited in-plane deformations. Peak displacements of the free column in each horizontal direction are always less than those in the bare structure under the same bidirectional excitation. More specifically, within the present range of variation in infill properties, peak displacement components of the free corner column of the infilled structure do not exceed 84, 89 or 52 per cent of the maximum unidirectional displacement of the bare structure, under the bidirectional motion E_x & E_y (i.e. that of the shaking table test), Acc.1 & Acc.2, or Acc.3 & Acc.4, respectively. These values compare fairly well with the corresponding ratios of 0·95, 1·02 and 0·7 estimated at the end of Section 3·2 from the associated spectra for an elastic period of 1·0 s for the infilled and the bare structures. It is to be remembered though that, according to equations (5) and (11), lumping of the masses at the four corners in the analyses increases the rotational inertia and reduces peak displacements by 22%, in comparison to the reality reflected by the test structure. If this difference is transferred to the results of the parametric analyses, the values of 0·84, 0·89 and 0·52 cited above for the ratio of peak displacements in the infilled and the bare structure become 1·02, 1·08 and 0·63, respectively, without changing the overall conclusion that the peak value of the displacement components are about the same in the bare and in the infilled structures.

A second set of parametric analyses is performed, this time regarding the post-ultimate softening ratio of the infills. In the base case considered in Section 4 the slope of the softening branch is equal to 1 per cent of the elastic, corresponding to a stable post-ultimate behaviour of the infills. Because in the base case the deformations of the infills do not exceed those at ultimate strength, these parametric analyses are performed for infill strength and stiffness reduced by a factor of 2·5 over the base case. Their results show that an increase in the softening slope from 1 to 2·5 per cent of the elastic branch does not affect appreciably the response.

In a final set of parametric analyses, the cross-section of the columns of the test structure are increased from 0·25 by 0·25 to 0·40 m by 0·25 m, and the three 12 mm bars along each one of the

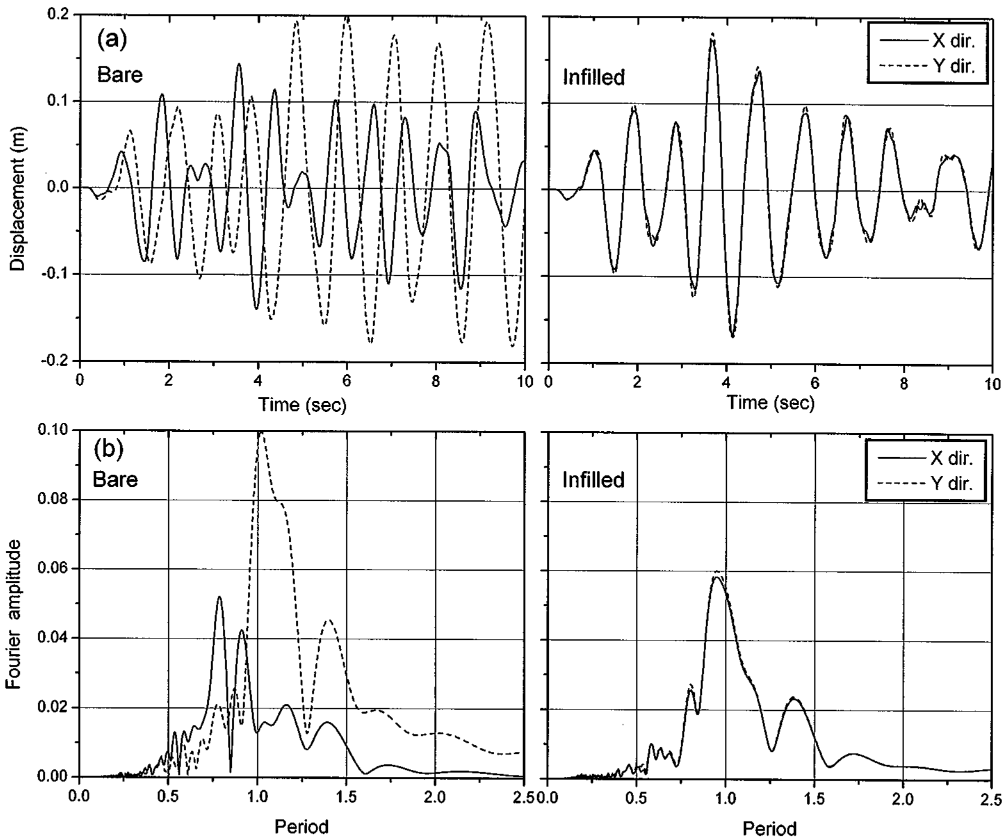

Figure 6. Top displacement of square structure with columns about 2·50 times stiffer and stronger in the X-direction than in Y: (a) time-histories; (b) Fourier amplitude spectra

short sides are replaced by three 14 mm bars. In this way the flexural capacity of the columns in the XZ vertical plane more than doubles and their secant stiffness at yielding in antisymmetric bending almost triples. Column capacity and effective stiffness in the XZ planes change very little (by 25 and 5 per cent, respectively). The so-modified structure is subjected as bare or infilled to the bidirectional time histories of the test E_x and E_y. Time histories of top displacements of the bare and the infilled structure are shown in Figure 6(a), while their Fourier amplitude spectra are compared in Figure 6(b). The predominant period of the response of the bare structure is 0·79 s in the strong direction (X), and 1·02 s in the weak (Y), while that of the infilled structure is 0·95 s, i.e. in between, in accordance with equation (9). Peak displacements of the top of the free-column of the infilled structure are equal to 0·18 m and take place simultaneously in both directions, while those in the bare are equal to 0·14 and 0·20 m in the X and Y directions, respectively. Peak displacements of the infilled structure are again of the same order, and indeed less than those of the bare structure in the corresponding direction.

6. CONCLUSIONS

A shaking table test on a two-storey, square in plan frame structure, with two adjacent sides infilled and the two other open, along with non-linear dynamic analyses of the response and a series of numerical sensitivity studies, have all shown that the response of such a torsionally unbalanced structure to a bidirectional ground motion with two independent components conforming to the same spectrum, is essentially rotational about the common corner of the two infilled sides, with predominant period about the same as that of the bare structure in the two horizontal directions X or Y. Moreover, the peak displacements of the free column in each horizontal direction are about the same as, or smaller than, those of the bare structure, but take place simultaneously. The implication for the design of RC frame structures with such plan-eccentric infilling, is that the columns near the common corner of the open sides should be proportioned for simultaneous occurrence of the full peak force or deformation demands due to both horizontal components of the seismic action. In all other aspects the RC frame structure can be designed as if it were bare.

If the results of the shaking table tests and of the inelastic dynamic analyses can be considered as a confirmation of the simple idealization of the response as a twisting motion about the common corner of the two infilled sides and of the associated analytical results in Section 3, the confirmation may be extended to the comparison between infilling of two adjacent sides or of one side only. If this is the case, the flexible-side RC elements of the structure with only one side infilled are much more critical under bidirectional motions, or under unidirectional parallel to the infilled side, than any of the RC elements of the structure with the two adjacent sides infilled. Moreover, they are not covered by the design of the structure as a bare frame. It seems, therefore, that although the doubly eccentric layout of infills in plan is sometimes quoted as the most extreme case of plan-wise infill irregularity, the infills in the transverse direction have an important beneficial effect on seismic response and performance under both unidirectional and bidirectional motions. This is consistent with conclusions of several recent studies of torsionally unbalanced inelastic bare structures.[3,5,11]

A secondary conclusion of the combined experimental and numerical work refers to the performance of infill panels under bidirectional excitation. This problem can be studied experimentally only by shaking table testing, such as that of the present study. The test results show that infill panels with a clear height of 2·5 m and as thin as 115 or 80 mm, can sustain lateral accelerations around 1·75 or 1·3 g, respectively without out-of-plane expulsion or significant damage.

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